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LAIDLAW WASTE SYSTEMS INC.

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May 14, 1992

Mr. Jim Hull
Director, Solid Waste Management Program
MDNR
P.O. Box 176
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Mr. Leon Golfin
St. Louis County DCHMC
111 S. Meramec Ave.
Clayton, MO 63105

RE: MAJOR MODIFICATION SUBMITTAL - BRIDGETON SANITARY LANDFILL GAS EXTRACTION SYSTEM

Dear Sirs:

Laidlaw Waste Systems (Bridgeton), Inc. is pleased to submit two (2) sets of reports and designs for installation of a Landfill Gas Extraction System. This submittal is intended for your review and approval, for installation at the Laidlaw-Bridgeton Sanitary Landfill. This report replaces the March 20, 1992 submittal. Design changes have been incorporated following comments made by the Regulatory Community.

If I may be of any assistance regarding this submittal, please feel free to contact me at (800) 288-2909.

Sincerely,

Laidlaw Waste Systems, Inc.

Miles Stotts
KS

Miles Stotts
Regional Environmental Manager

enc.

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Design Submittal (Revised)

Landfill Gas Extraction System
Laidlaw Waste Systems (Bridgeton), Inc
Bridgeton, Missouri

May 18, 1992

I PROCESS OVERVIEW

Landfill gas (LFG) is generated by the anaerobic decomposition of refuse buried in the landfill. LFG consists mainly of methane (45 - 50%), carbon dioxide (45% - 50%), trace amounts of organic compounds, and sulfur bearing compounds. The methane content of LFG makes it a very good fuel source. LFG is currently being used at many landfills for either electrical generation or as a replacement fuel for natural gas in industrial equipment. Laidlaw Waste Systems (Bridgeton), Inc, is applying for the installation of a LFG flare, but LWS is pursuing useful energy recovery for this site.

The existing, permitted LFG extraction and ventilation system will be upgraded to provide active gas extraction from active and closed portions of the sanitary landfill. Extraction will be achieved via header connection to fourteen (14) existing wells, four (4) new wells, and three (3) new horizontal trenches to dual multi-staged gas blowers. The extracted LFG will be incinerated by a 2500 standard cubic feet per minute (SCFM) capacity, IT McGill enclosed flare. Please see Section IV for initial LFG production estimates.

II PROJECT HISTORY

In June 1985, on behalf of Westlake Sanitary Landfill, Burns & McDonnell (Kansas City, MO) submitted to MDNR, a permit application which included designs for the currently installed LFG venting system. This venting system encompasses the existing four (4) gas vents (GC # wells), and four (4) leachate collection wells (LCS # wells) in the current fill area.

In or around 1986, Westlake permitted and constructed a LFG flare, located at the end of Taussig Road. This system extracted LFG from six (6) wells located in a previously filled area north of the flare.

In 1989, LWS made repairs to the header piping and the six LFG extraction wells. Safety and operational conveniences were added to the flare and blower controls.

In 1991, LWS began to receive complaints concerning odors around the landfill. A subsequent meeting with all responsible regulatory agencies was held in December 1991, and LWS proposed a remediation schedule. This report and accompanying design will achieve the results agreed upon during that meeting.

Subsequent to that meeting, LWS employed the use of odor masking agents in the active area.

These agents will continue to be used throughout installation of the proposed LFG extraction system.

III DESIGN NARRATIVE

A) Extraction wells in closed portions of landfill

Four (4) new, LFG extraction wells will be installed in the area southwest of the existing flare station. These wells will be installed to a depth of approximately (60) feet, using a truck mounted, 36" bucket auger. These wells will be constructed with High Density Polyethylene (HDPE) or Poly Vinyl Chloride (PVC) piping. Detail 1-4 shows typical LFG well construction data.

When landfill gas is collected, water vapor in the gas cools and forms condensate. This condensate must be drained from the collection header so that is does not pool and form blockages. The collection header at the Bridgeton landfill will be sloped to condensate lift stations.
New HDPE header piping of various diameters will be installed to carry the LFG to the existing flare station. The header will be installed in such a manner to convey both LFG and liquids (condensate) to a condensate lift station (Detail 2-4).

At the condensate lift station, the collected liquids will be "pumped" to a holding tank located at the "new" flare station. This condensate will be handled as leachate, and transferred to the existing leachate aeration lagoon. Following treatment, this will be discharged to the municipal sanitary sewer system.

A six (6) inch HDPE header will be constructed along the existing six (6) wells. Prior to installation of this portion of the header piping, dynamic consolidation may be utilized to stabilize the refuse and soils surrounding the header. This dynamic consolidation is intended to minimize the effect of differential settlement around the header. Dynamic consolidation has been successfully used at other municipal solid waste landfills, reports of which are in Appendix 1. To promote correct surface water drainage in this area, LWS may place additional refuse capacity in this area. Any additional refuse would be placed within currently permitted areas, and would not exceed permitted vertical contours.

As with the other header, a condensate lift station will be installed to remove liquids from the header piping.

B) Extraction trenches

A series of three (3) LFG trenches will be installed in the area of the landfill currently known as the "wet weather area". The trenches are designed to allow removal of LFG, while allowing further filling to occur. These trenches will be constructed of eight (8) inch perforated HDPE or PVC, bedded with two (2) feet of suitable, granular material, as shown in Detail 3-3.

These trenches will be connected to HDPE header piping by use of a collection riser (Detail 7-3). Each collection riser will be equipped with butterfly valves to control gas flow from the trenches. As with the other header piping systems, a condensate lift station will be installed to collect liquids.

C) Active area extraction system

LFG extraction will occur by use of the existing gas wells (GC-1, GC-2, GC-3, and GC-4), and leachate collection wells (LCS-1, LCS-2, LCS-3, LCS-4). Both the gas and leachate wells will be connected to HDPE header piping as shown on detail 6-3. Each collection point will have a butterfly type valve to control LFG flows to the header. Additionally, these valves can be used to isolate individual wells from the header system, to facilitate filling of refuse around the well. To monitor LFG content and header vacuum, monitoring points will be installed at each well.

Air intrusion will be limited by use of mechanical seals installed in each collection well. This mechanical seal could include plates, bladders, or caps. LWS is currently evaluating different types of seals, and will choose the most appropriate type.

The header system will consist of both trenched an non- trenched HDPE piping. That is, certain portions of the header piping in the active area will be installed above grade. This is required for expanding the system as landfilling continues. Equipment crossing areas will be installed as shown in Detail 7-4.

Condensate from the header piping will be gravity drained into the four (4) leachate collection wells in the active area.

The specific design of the header system may be altered during actual installation to facilitate current landfilling and traffic patterns. Any changes will not alter anticipated performance or reliability of system.

D) LFG flaring station

The LFG Management Facility will be the final collection ppoint for all the LFG. Two (2) Multi-staged blowers, which provide the vacuum, and filtration equipment to separate the free water in the gas, will be located at this facility.

Prior to being drawn into the blower, the LFG will pass through a water separator. the separator is designed with an interior baffle to collect the free water in the gas, and direct it to a drain. Water from this vessel, as well as that form the low point of the gas header, will be collected in a tank and transferred via tanker truck to the existing leachate aeration lagoon.

After passing through the separator, the LFG will be drawn into one of two blowers, which will pressurize it to approximately 1 psig for delivery to the flare. The blowers will be a multi-staged centrifugal unit with an internal coating to resist the corrosive nature of LFG. Each will be capable of moving 1250 cubic feet per minute of LFG, with an inlet vacuum of approximately 50 inches of water and a minimum discharge pressure of 15 inches of water. Surge protection will be installed to de-energize the blower if gas flow falls below design.

A pneumatic valve will be located between the blower and flare. This valve will open when signaled to do so by the flare. The valve will be such that it will fail close if there is a loss of power, or if the flare detects a loss of flame. This feature will prevent the venting of unburned LFG out of the flare.

The flare will be an enclosed flame ground flare, currently the best available technology for LFG and digester gas operations. These flares are equipped with automatic controls for safe start-up and shutdown. A propane pilot is used to ignite the LFG. This pilot will relight if a flame- out of the LFG should occur. If the LFG flame cannot be re- established, the controls will close the spring loaded valve and also shutdown the blower. The flare controls will then sound an alarm to notify landfill personnel that the system is down.

The flare will have a maximum capacity to burn 2500 scfm of LFG with a methane content of 50% and a minimum capacity of 750 scfm at 20% methane. A flame arrestor will be provided with the flare to prevent any potential backfire from progressing past the flare base.

IV LFG Production Estimates

LFG production is dependent on several site specific factors of the buried refuse including its' age, composition, and moisture content. LFG production estimates for the Bridgeton landfill are based on field data obtained from landfills accepting municipal solid waste and having similar moisture contents.

In dry climates, refuse produces LFG at approximately 0.08 cubic feet per pound of refuse per year (cf/lb-yr). In wetter climates, refuse produces LFG between 0.1 to 0.15 cubic feet per pound of refuse per year (cf/lb-yr). For the Bridgeton landfill, a value of 0.15 cf/lb-yr is used.

To calculate the weight of refuse each well will influence, it is assumed that the area of influence for each well is a square with a side of 150 feet. The depth of refuse for each well was then found by determining its' current elevation minus the elevation of the bottom of the refuse cell. It is assumed that the refuse density throughout the landfill is 1200 pounds per cubic yard of volume.

To calculate the amount of gas from a well located in refuse an average of 60 foot deep:

((150 ft x 150 ft x 60 ft x 1200 lbs/cy) x 0.15 cf/lb-yr/(27 cf/cy x 525600 min/year)) = **17.1 cubic feet per minute at standard conditions (SCFM)**

A horizontal collection trench will produce a similar amount of LFG when additional fill is placed over it. For this calculation, it is assumed that the area of influence will be 150 ft (horizontal) by 40 ft (vertical). The trench will have approximately 390 ft of perforated pipe, therefore productions estimates are as follows:

((150 ft x 40 ft x 390 ft x 1200 lbs/cy) x 0.15 cf/lb-yr/(27 cf/cy x 525600 min/year)) = **29.7 SCFM**

Due to perforated LFG collection piping radiating from the existing GC # wells, they act as both vertical and horizontal wells. To a similar extent, this is also true for the LCS # wells. To calculate the estimated LFG production from these wells, a radius of influence of 300 ft by 300 ft is assumed. For purpose of estimation, the typical well depth used is 100 ft. Therefore productions estimates are as follows:

((300 ft x 300 ft x 100 ft x 1200 lbs/cy) x 0.15 cf/lb-yr/(27 cf/cy x 525600 min/year)) = **114.2 SCFM**

Initially, the following amount of LFG is expected to be collected from the Bridgeton landfill:

10 vertical wells in inactive areas @ 17.1 SCFM	=	**171 SCFM**
3 horizontal collection trenches @ 29.7 SCFM	=	**89 SCFM**
8 existing wells in active portion @ 114.2 SCFM	=	**914 SCFM**
	Total (est.)	**1174 SCFM**

A measuring device (orifice plate or pitot tube) will be installed at the flare station to provide the actual operating LFG flow.

V Example LFG System Monitoring Plan

Appendix 2 contains a standard operating plan for LFG systems. Upon installation of this system, a specific plan will be written for the Bridgeton LFG system. This plan will be submitted with the as-built documentation.

VII Project Schedule

Design Submittal	May 18, 1992
Design Approval	June 1, 1992
Collection System Installation Completed	July 15, 1992
System Start-up	August 3, 1992
Submittal of As-built Documentation	September 1, 1992

APPENDIX 1

References on Dynamic Consolidation

Densifying a Landfill for Commercial Development

S. B. Steinberg
Project Engineer, STS Consultants, Ltd., Northbrook, Illinois

R. G. Lukas
Senior Principal Engineer, STS Consultants, Ltd., Northbrook, Illinois

SYNOPSIS This paper presents a case study of a dynamic compaction ("pounding") project, undertaken in Skokie, Illinois. The purpose was to densify a 50-ft deep former municipal waste landfill for support of a one-story warehouse structure on shallow foundations. The majority of the pounding was performed utilizing a 15-ton weight falling from a height of 60 ft. In some areas, lower energ levels were used for surface compaction. All phases of the project are discussed, beginning with the subsurface exploration program and geotechnical analysis, through the experimental test poundin section, and the final check borings to observe that the "production" pounding was successful. A follow-up of the performance of the pounding, by monitoring foundation settlements, is discussed, as are topics such as depth of improvement, offsite vibrations, and energy input.

INTRODUCTION

In 1980, a detailed subsurface exploration and geotechnical analysis were performed, establishing the proposed building site as a former clay pit filled with municipal waste. A cost/benefit analysis comparing available foundation alternatives was developed, and, as will be discussed, dynamic compaction, hereinafter referred to as "pounding", was considered to be the most cost-effective and acceptable option. Since the degree of improvement that could be attained could not be precisely predicted in advance, a pounding test section was completed. Standard Penetration and pressuremeter tests were performed before and after test pounding. Sufficient energy was applied until the fill was considered adequately densified to support the structure on shallow foundations. Throughout the course of the project, communication and coordination was required between the owner, the consulting and design engineers, and the contractor.

In this paper, all phases of the project are discussed, beginning with the subsurface exploration program and geotechnical analysis, through the final check borings to observe that the production pounding was successful. A follow-up of the performance of the pounding, by monitoring the foundation settlements, is also discussed, as are topics such as depth of improvement, offsite vibrations, and energy input.

PROJECT DESCRIPTION

The structure proposed for construction consisted of a one-story, slab-on-grade, 22-ft high steel frame building, 78,400 square feet in plan. Bay spacing was 40-ft by 40-ft. Interior column loads were on the order of 80 kips. The exterior wall load was approximately 6 kips per lineal foot. Floor slab loads were on the order of 400 to 500 pounds per square foot (psf). Truck loading docks were planned for the southern side of the building and on-grade passenger car parking was planned alongside the western part of the building. It was also the intention of the owners to construct a building which would have the necessary capacity for additio load on the northern and eastern sides. Thus, although one building was planned, the site densification process was to include areas of future expansion, as well as aforemen tioned parking and driveway areas. The combined project area was 102,400 square feet.

SITE CONDITIONS

A search into the history of the site using air photos and maps revealed that the propose building area was contained (with the excepti of the northeastern corner of the property) within the limits of a former and abandoned clay pit. On the basis of the soil borings performed, it was concluded that the pit depth within the project area ranged from approximately 40 to 50 ft at the deepest portion to 22 to 37 ft along the outer edges. Mining in the clay pit ceased around 1936 and the site was used as a municipal waste landfill until approximately 1950. The site was then used only as an outdoor movie theate parking lot. At one time, the fill heights extended approximately 5 to 15 ft above the surrounding street grade. The excess fill was removed approximately one year prior to initiating the pounding.

The fill consisted of varying amounts of decomposed refuse, cinders, ashes, brick, and occasional pieces of wood and organic matter. Broken concrete, paper, and glass bottles were also encountered. A grain size analysis performed on the fill indicated a material similar to a sandy, fine to coarse gravel with little silt and clay (GP-GC). Based upon the Standard Penetration Test (SPT) and the pressuremeter test results, the fill was observed to be generally in a loose to medium dense condition. In some pockets of extremely loose fill, the SPT values were less than 5 blows per foot. In other instances, no resistance was met by the pressuremeter probe. Generally, water content values in the fill were between 20 and 30%, with occasional organic pockets exhibiting water contents as high as 65 to 95%.

Generally, the fill materials were underlain by medium stiff to stiff, natural silty clay. With depth, the consistency of the clay increased from very stiff to hard. "Hardpan" soils (typical to the Chicago area) which are identified by low water contents and high strengths, were encountered at depths varying from 56 to 60 ft below the existing ground surface.

The ground water table was located at a depth of approximately 5 to 8 ft below the existing ground surface.

FOUNDATION SELECTION

Following identification of the subsurface soil and ground water conditions at the site, the options available for foundation support of the proposed warehouse were identified. Initially, three (3) options were considered. Ultimately, two (2) of the options were ruled out in favor of the third. Prior to eliminating two of the options, a comparison of cost as well as construction feasibility and timing was undertaken.

Option 1 -- Dynamic Compaction

Because of the erratic thickness and composition of the fill, it was imperative that if any new construction was to be supported directly on the fill, site densification would be required. Fortunately, the fill had been in place a sufficient amount of time, and the majority of the organic material appeared to have decomposed. No gas was detected as the bore holes were advanced. Thus, concern over significant future decomposition and resulting settlement was not a consideration.

It was recommended that a suitable solution for densifying the soils would be by means of pounding to a point where spread footings and a slab-on-grade system could be utilized. Based upon the Menard (1975) formula modified by Leonard, et al (1980) and Lukas (1980), a 15-ton weight dropping a distance of approximately 60 ft was determined necessary to achieve proper densification. Once the area had been densified, the footings could be placed within the fill and designed for a maximum net allowable soil bearing pressure of 3,000 psf. Previous experience with densification of landfill deposits by pounding indicated that a pressuremeter modulus of 50 tsf within 10 ft of ground surface and 30 to 40 tsf at lower levels would be achieved. For the magnitude of the loads, this would result in a predicted settlement of approximately 1 inch.

Option 2 -- Deep Foundation Alternative

The second option consisted of extended foundations and a structural floor slab. The most suitable foundation, given the possible corrosion potential of the fill, as well as the proximity of existing structures, was caissons (drilled piers) extending to the "hardpan" soils at a depth of approximately 56 ft below existing grade. At this depth, the caissons could be designed for a maximum net allowable soil bearing pressure of 20,000 psf.

Several drawbacks to the caisson foundation alternative were anticipated. These included the need for permanent steel casing through the fill materials and soft clay layers to prevent sloughing, caving in, or squeezing of these materials into the shaft excavation. The casing would increase the cost of the project and the construction time. It was also anticipated that the contractor may encounter obstructions from large concrete chunks in the fill which would add further to construction costs and delays.

Option 3 -- Combined System

The combined system involved dropping a lighter weight, such as an 8-ton weight, and densifying the upper portion of the fill for support of the floor slab. Deep foundations would still be used to support the structure. This option would reduce the building settlement which would be encountered in Option 1, while at the same time alleviating the necessity of a structural slab which would be required for Option 2. However, the high cost of caissons was still present, as was the possibility of construction delays.

Cost Analysis

A cost analysis of the first two options was performed. A price for the third option was not prepared, since the anticipated delays with the combined system were not tolerable. The client was very concerned that the occupancy date be met. The anticipated costs were as follows:

1. Pounding to densify area, stone necessary to raise site to design subgrade, and cost of slab and footings $500,000
2. Caissons/structural slab $1.0 - $1.3 million

On the basis of cost, construction feasibility, as well as construction timing, Option 1 was selected, with the understanding that an experimental test pounding section would precede production pounding.

MONITORING PROGRAM

Depending upon the soil type encountered, representative soil samples in the check borings were obtained by means of the split-barrel and shelby tube sampling procedures. However, settlement and bearing capacity evaluations were based on the pressuremeter tests performed at selected test depths. Due to the larger test area in the pressuremeter device, a more representative evaluation of the compression characteristics of fill can be obtained. This is particularly important in erratic fills, since the pressuremeter test averages out inconsistencies to obtain representative values.

In addition to the aforementioned testing, three field monitoring methods were employed. These were: full-time field inspection by a qualified soil engineer familiar with the pounding process; continual checks of crater depths; and monitoring of overall ground settlement after each leveling pass. Through experience, Lukas has found that the overall ground settlement following pounding and leveling can be expected to be on the order of 10% of the total depth of improvement in the fill. Thus, continual monitoring of the site settlement is an indication of the effectiveness of the pounding. The purpose of monitoring crater depths is to isolate inconsistencies for further evaluation. Hard spots can be an indication of a crust forming, and soft spots can either be an indication of unsuitable soils which should be removed or of an area where additional pounding is required. Both services require inspection by full-time field personnel who are familiar with the pounding process and have the experience and authority to alter procedures when necessary.

Ground velocities developed during pounding were monitored at increasing distances from the drop location. These results indicated that the surface vibrations in the fill rapidly dampened with increasing distance from the point of impact. An analysis was also made as to how the monitored vibrations compared with vibrations measured in other soil types. A graph depicting this comparison is shown on Figure 1. The vibration monitoring was also used as a guide in determining the effect of the pounding on adjacent live utilities. The pounding came within 15 ft of buried utility lines with no damage occurring. In addition, no damage was observed to adjacent structures. The vibration monitoring was performed utilizing a VME seistector which measured resultant peak velocities.

TEST POUNDING

For the test pounding, a 60-ft by 60-ft area was selected. The section was located in the vicinity of a boring which indicated the thickest (50 ft) and potentially loosest deposit of fill. A 15-ton weight, manufactured by the contractor, consisted of a series of horizontal steel plates bolted together to form a cylindrical shape. A 6-inch thick bottom plate was attached to the weight to

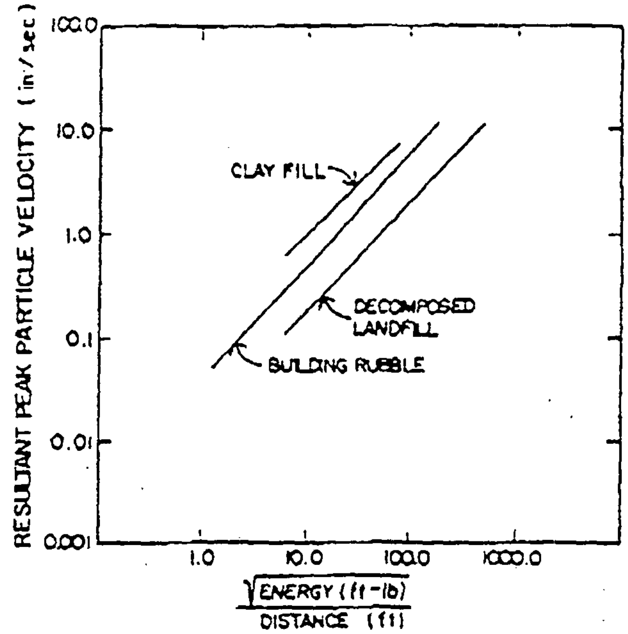

Fig. 1. Resultant Peak Particle Velocity vs. Energy Input and Distance from Weight

facilitate extraction from the fill and reduce suction forces. The diameter of the bottom plate, slightly larger than the remaining portions of the weight, was 5 ft. The design contact pressure was 1,530 psf. The remaining dimensions were 4'7" high and 4'6" in diameter. In order to lift the weight 60 ft in the air, a 100-ton capacit crane was required.

Eight-foot crater spacings were selected and a total of 93 individual craters were created in the test section. The pounding was actually completed in two phases, with the first phase consisting of 54 craters across the entire site on the 8-ft grid, followed by an additional 39 craters superimposed over pounded and regraded surface area. Each crater received an average of two to three tamps per location per pass. Crater depths averaged approximately 6 ft, and an attempt was made to keep the weight penetration above the water table. Originall three passes were planned in the test section, but difficulties with the crane equipment prevented this within the time budget for the test pounding. Both crater depths and adjacent ground heave were carefully monitore throughout the test pounding process. The average energy input was approximately 56 tons-ft/ft^2 (184 ton-m/m^2). Average groun reduction following the test pounding was 3'6", amounting to approximately 11% of the total depth of improvement (30 ft).

A soil boring was performed in the test secti following the pounding and SPT and pressuremeter test results were obtained. These results were compared to tests performed prior to pounding and a graph comparing the results is shown on Figure 2. It was interesting to note that an increase in density was observed immediately above the natural clay.

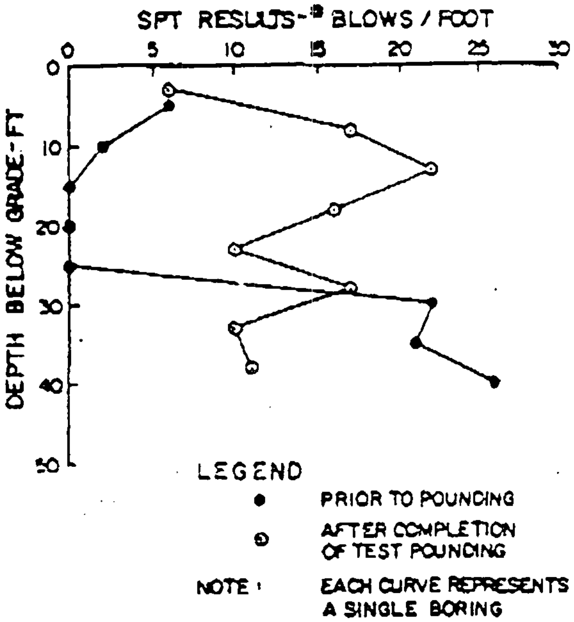

Fig. 2A. Standard Penetration Test Results Before and After Test Pounding

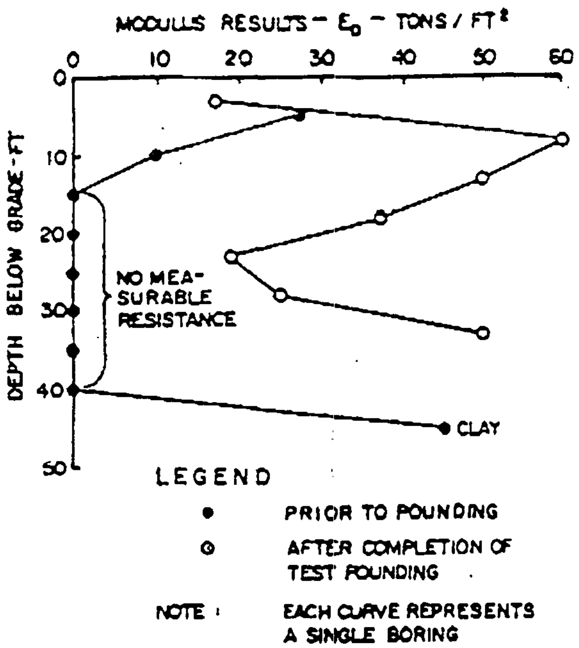

Fig. 2B. Pressuremeter Test Results Before and After Test Pounding

On the basis of the test pounding results, it was concluded that a 15-ton weight dropping from a height of approximately 60 ft would be utilized. Crater spacings would be on the order of 8 ft (center to center). Three to five passes, with each pass involving a minimum of two tamps, or that number necessary to achieve a maximum crater depth of 6 ft, was specified. Following each pass, the craters were to be leveled. Following the final pass, the craters were to be releveled and the surface compacted. With regard to minimum density criteria, it was suggested that an average minimum of 15 blows per foot be achieved with the SPT test and an average minimum modulus of 50 tsf be achieved with the pressuremeter test within 10 ft below the footing and 30 tsf below this level.

PRODUCTION POUNDING

Construction Difficulties

Despite the success of the procedure as indicated by the test section, several construction-related difficulties were encountered during the production pounding phase. One of the most significant problems was related to the consistency of the fill. Although the material noted by the soil borings appeared to consist primarily of miscellaneous rubbish and building materials, isolated pockets of organic and clayey soils were encountered. These areas were so soft that crater depths oftentimes averaged 7 to 10 ft on a single drop. At these depths, the weight became very difficult to extract, due to suction forces which developed. Some deep craters were also encountered in the more granular portions of the site, but suction was not as dramatic in these soils and extraction of the weight was typically a routine process. To deal with the suction problems in the more cohesive soils, the following procedure was established:

> On the first pass across a new area, the weight was dropped one or more times per crater, depending upon the resistance offered. The object was to produce a crater less than 7 ft deep. Typically, five or six passes were necessary to achieve the required density. If the soft and wet areas contained predominantly cohesive soils, the materials were removed to a depth of about 7 ft and backfilled with granular soil. Large stone was recommended for deep undercut areas, while smaller size stone was recommended for the upper 3 ft of new fill placed.

A second problem which appeared was the loose, fluffy material which collected at the ground surface. Even with the second and third passes of the weight, it was difficult to compact these surface materials to a point where they were suitable for a floor slab subgrade. Thus, the procedure developed was that once the final pass had been completed, crushed rock was used to fill the craters and to bring the area to approximate floor slab subgrade. The stone was compacted either using the 15-ton weight dropping a distance of 20 ft, or a lighter weight (5 to 6 tons) dropping a distance of 30 to 40 ft. It was found that the lighter weight was the preferable solution since a secondary crane was brought out to do the surface tamping and production with the larger crane was not slowed.

Water created a problem with the deeper crater depths, and also became a problem when the pounding approached the pit edges. This problem occurred as a result of the change in material type from the fill to the natural clay soils. In effect, the pounding forced the water towards the pit edges, but the lower permeability of the clay slowed the water passage. The consequent build-up of water in the craters reduced the pounding effectiveness. The solution to this problem was to cut isolated drainage trenches into the sides of the pit, thus providing an exit point for the water. Water was continually pumped out of these trenches and the ground water was subsequently lowered in the immediate pit area.

Another unusual problem which occurred was associated with the longevity of the cables used to lift the 15-ton weight. At first, the contractor utilized a 7/8-inch diameter cable. However, the cable broke on the average of once every two days. Thicker cable could not be utilized due to the mechanical restrictions of the crane. Finally, it was decided to use a 1-1/8-inch diameter cable reduced during fabrication to 7/8-inch. This was a relatively successful solution, although mechanical breakdowns still did occur.

Summary of Results

Soil borings with pressuremeter and SPT tests were performed following completion of the production pounding. An averaged comparison of before and after data is presented on Figure 3. From the data, as well as our full-time observation during the production pounding, it was concluded that sufficient compaction was achieved.

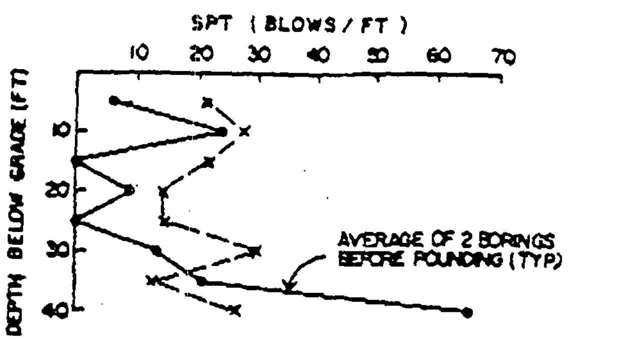

Fig. 3A. SPT Results Before and After Pounding

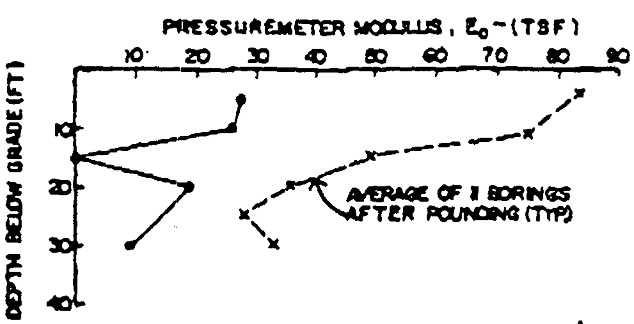

Fig. 3B. Pressuremeter Results Before and After Pounding

As with the test pounding phase, the depth of improvement was observed to be on the order of 30 ft. This depth of improvement (in equivalent meters) computes to be approximately $0.58\sqrt{WH}$, where W = 15-ton weight and H = 18-meter drop. A comparison was also made between areas where pounding was performed on stone and no stone surfaces. The density improvement appeared slightly greater where stone was placed in the craters and on the surface prior to the final phases of tamping. Stone thicknesses were on the order of 3 to 4 ft. On the basis of an average of twelve tamps at each location, the average energy input with the 15-ton weight was 170 ton-ft/ft^2 (560 ton-m/m^2). Ground subsidence after pounding, and releveling and compacting, averaged 3.5 to 4.0 ft, which was approximately 12 to 13% of the total depth of improvement of approximately 30 ft.

Long-Term Performance

In order to evaluate the long-term performance of the structure, settlement markers were established on the building and were monitored for a period of six months from footing construction through completion of the superstructure. A summary record of the results is shown on Figure 4. Readings of initial settlement were only slightly higher than the predicted range of an inch. Long-term settlement was less than 1/4 inch. To date, no known signs of distress have occurred to the building.

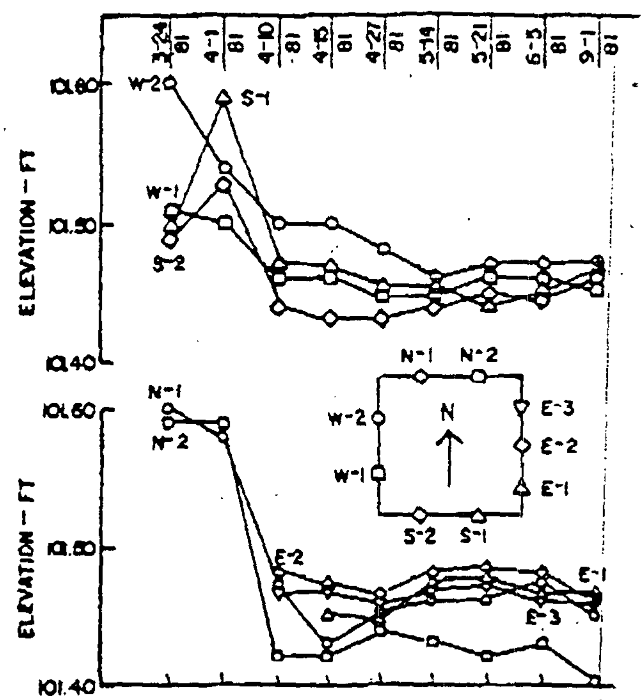

Fig. 4. Settlement Record

CONCLUSIONS

Given the site conditions, the age of the fill, and the extent and density of the fill, the pounding alternative was considered the most cost-effective solution for the site. On the basis of the long-term performance of the building, indicating maximum settlements within the predicted ranges, the pounding process was considered a successful alternative. In the process, several new construction techniques were learned. These were associated with winter weather difficulties, cable problems, water removal, the importance of the pounding and grid sequence, as well as the use of stone stabilization to facilitate surface tamping. Coordination between all parties became a critical factor, as did full-time inspection during construction. In summary, marginal sites such as former landfills can be successfully and economically developed if properly evaluated and carefully monitored during construction.

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Densification of a decomposed landfill deposit

Densification d'un dépôt d'ordures ménagères dans lequel le processus de décomposition de matières putréfiables est complet

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SYNOPSIS A decomposed municipal waste landfill deposit was improved by different compaction techniques for support of a regional shopping center, consisting of one and two-story buildings plus surrounding parking lots and driveways. In the fill areas, the landfill was compacted to adequate densities by conventional earth-moving and compacting equipment. In the cut and transitional areas, the load-bearing areas were improved by means of dynamic compaction. The improvement was sufficient to limit settlements of footings with column loads ranging from 1,000 to 1,500 kN to values on the order of 1 to 2 cm.

INTRODUCTION

During 1975 to 1977, a regional shopping center was constructed in Chicago on a former landfill deposit. The buildings consisted of one and two-story reinforced concrete structures with column loads on the order of 1,000 to 1,500 kN. The shopping center occupied a 215,000-m^2 site. The landfill deposits were compacted by conventional earth moving equipment in the fill areas and densified by means of dynamic compaction in the cut and transitional areas. The depth of landfill below final grade ranged from 9 to 18 m. The buildings were constructed with footings supported on the landfill and with slabs-on-grade.

PRE-CONSTRUCTION CONDITIONS

Beginning in 1919, natural clay was removed from this site and used for making bricks. Over a period of approximately 30 years, this resulted in a lowering of the site to depths ranging from 15 to 18 m below surrounding street grade. The deep excavation was filled with refuse in an uncontrolled manner. The refuse consisted of municipal waste, demolition debris, wood products from removal of dead trees, and miscellaneous materials. Open burning occurred in the pit at isolated locations from heat generated during the decomposition. The landfilling operations ceased in 1947 and left a profile similar to that shown on Figure 1. At the south end of the site, the landfill was piled to a height of about 37 m above street grade while at the north end of the site, the pit remained open. Leachate accumulated in the lower portion of the pit and was periodically pumped into the city sewer system.

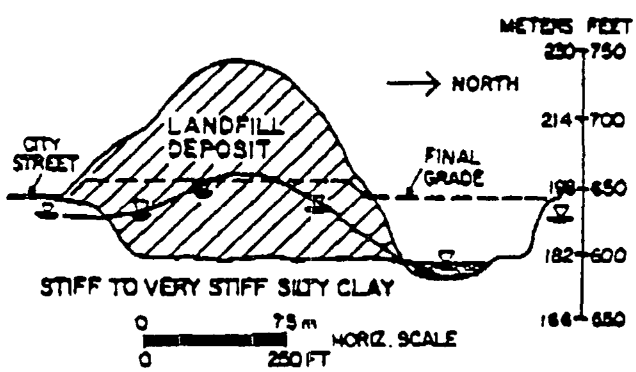

FIGURE 1 - GENERALIZED SUBSURFACE PROFILE

SITE PREPARATION

In order to balance the cut and fill operations, the surface elevation for the shopping center development was set at street grade near the north end of the project site and approximately 6 m above street grade at the southern end. Approximately one-million cubic meters of landfill was removed from the southern end of the site and filled into the northern end. The fill material was hauled by conventional earth scrapers and compacted in maximum lifts of 30 cm. The compacted fill was intended to form a suitable subgrade for building and roadway construction. At the southern end of the site and in the transition areas between the cut and fill, dynamic compaction was utilized to densify the upper portion of the landfill deposit. The most crucial zone for densification was the transition area because the deposits in this region had not been pre-loaded by the height of the former landfill, nor was any compaction applied during filling operations.

DESCRIPTION AND PROPERTIES OF THE LANDFILL DEPOSITS

At the time of construction, the landfill had decomposed to a material that can best be described as a well-graded granular material containing fines, making the soils slightly cohesive. The grain size gradation of a typical sample is shown in Figure 2. Approximately 40% of the sample is in the sand-size range with 20% each of silt and gravel. About 15% is classified as clay. In addition, there were large chunks of concrete, occasional timbers, numerous bottles, rubber tires, and pieces of metal in the fill deposit. No organic materials such as paper or rubbish were identified within these deposits. Atterberg limits tests performed upon the portion passing the No. 40 sieve yielded a liquid limit of 31 and a plasticity index of 7. According to the Unified Soil Classification System, this

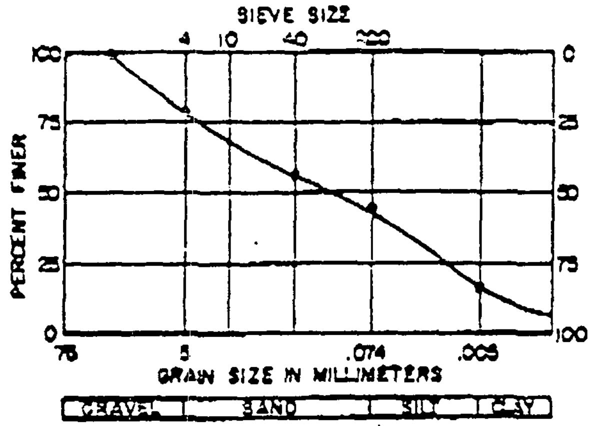

FIG. 2 - GRADATION OF LANDFILL SPECIMEN

specimen would be SC bordering on SC-SM. The natural water content of the fill was on the order of 15 to 27% above the prevailing water table and about 20 to 30% below the water table.

Monitoring of boreholes with gas measuring devices indicated that combustible gas was not present within the landfill which is another indication that the organic matter had decomposed. A gas venting system that was originally thought necessary was not provided beneath the slabs.

The Standard Penetration Resistance Values were quite erratic and were frequently as low as five blows per 30 cm to depths as great as 20 m below grade. Higher Standard Penetration Resistance Values were encountered at certain levels, presumably on larger obstructions within the fill.

Pressuremeter tests were performed within boreholes extending through the fill. Typical limit pressure values plotted against depth are shown on Figure 3.

The tests shown by the solid dots represent the tests performed in advance of construction. Most of the tests fall within a band labelled "Preconstruction Limit Pressure Range".

This range indicates the self-bearing limit pressure for this deposit; i.e., the limit pressure for a normally consolidated landfill deposit at varying depths below grade. This band represents a lower bound of limit pressure from which to compare the limit pressure of compacted fills. Compaction will induce preconsolidation of the fill deposit so the limit pressure of compacted fills should be higher than this range. The difference between the limit pressure of the compacted fill and the lower bound range will depend upon the degree of compaction achieved. Previously, the self-bearing limit pressure for a sand has been reported as 6 bars (1) and for a silt as 4 bars. Unfortunately, these values were not correlated to the confining pressure of the overburden.

Undisturbed specimens of the landfill could not be obtained with shelby tube piston samplers because of the large size debris within the landfill as well as the extremely loose condition of this deposit. To obtain additional information on the shear strength parameters of densified landfill, two large diameter (30 cm) specimens were compacted and tested in a triaxial chamber. The specimens were compacted to a unit dry weight of 16.9 kN/m^3 which corresponds to 95% of the

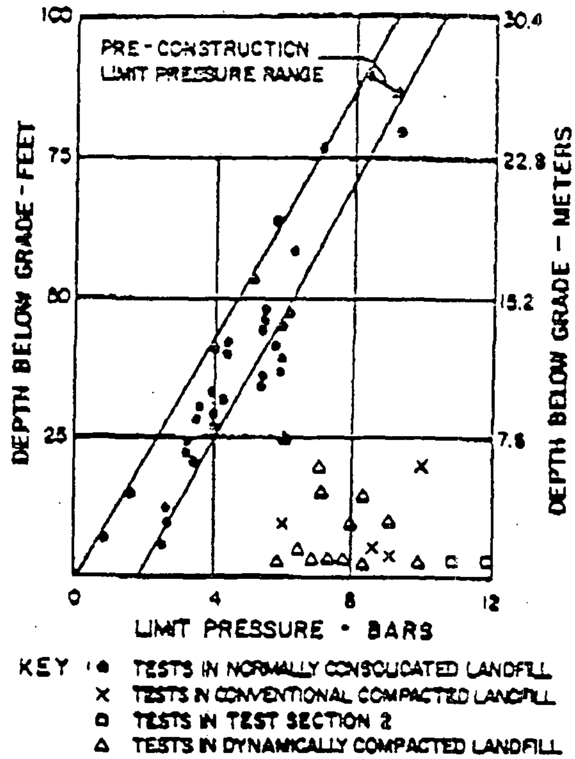

FIGURE 3

PRESSUREMETER TEST RESULTS

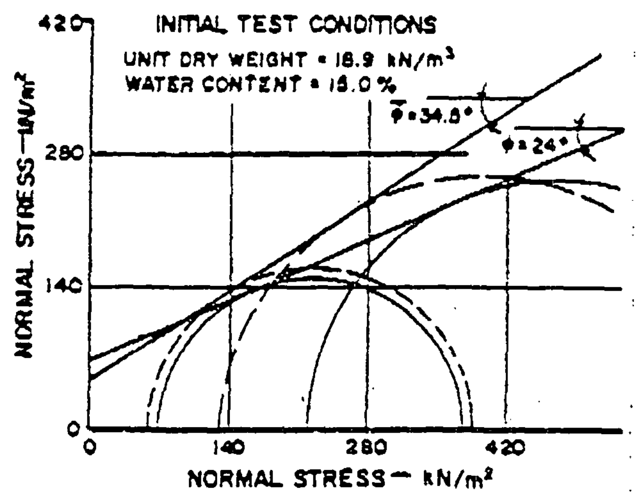

FIGURE 4

CONSOLIDATED UNDRAINED TRIAXIAL SHEAR TEST ON LABORATORY COMPACTED LANDFILL

maximum density obtained in the field test rolling discussed in the next section of this paper. The specimens were compacted at the natural water content of the fill which was 15%. Each sample was then saturated by the backpressure method prior to performing a consolidated undrained triaxial, $\overline{CU}$ test with pore pressure measurements.

The results of the $\overline{CU}$ triaxial texts are shown in Figure

The drained angle of shearing resistance, $\bar{\phi}$, was 34.5° with a cohesion intercept, $\bar{c}$, of 39 kN/m^2. The friction angle, $\bar{\phi}$, is typical for a granular soil containing a significant portion of silt (2). The cohesion intercept is higher than anticipated and is attributed to the clay size particles within the landfill.

CONVENTIONAL COMPACTION RESULTS

Because of the erratic nature of the fill and high amount of large size particles, laboratory moisture-density tests were not performed. The maximum unit dry weight for compaction purposes was determined in advance of construction by field compaction test sections. Three test sections were initiated to determine the compaction characteristics of the landfill deposits at different locations. Site material was spread out to a thickness of 30 cm over a width of 6 m and length of 15 m. At each test section, one portion of the strip was compacted with a self-propelled vibratory roller. The remaining portion was compacted with a 104-cm diameter sheepsfoot roller pulled by a dozer. Three lifts of site materials were placed and compacted at each test section.

All three test sections behaved in a similar manner, so only the test results from test section 2 are shown in Figure 5. The vibratory roller resulted in a unit dry weight of 17.8 kN/m^3 after five passes. The sheepsfoot roller resulted in low and erratic compacted unit weights ranging from 13 to 14 kN/m^3. The feet of the compactor tended to loosen the deposit as they lifted from the landfill which was detrimental to obtaining good compaction. The vibratory roller was then selected for the construction. In the future building areas, a minimum compacted density of 95% of 17.8 kN/m^3 was established. In future parking areas, the specified compaction was 90% of 17.8 kN/m^3.

Three pressuremeter tests were performed at a depth of 0.6 m below grade within the compacted test section. The pressuremeter modulus was found to range from 120 to 130 bars with limit pressures ranging from 11 to 13 bars. This represents a vast improvement over the "in-situ" conditions. As shown in Figure 3, a comparable limit pressure of 11 bars would not be attained within a normally consolidated landfill unless the overburden were greater than about 30 m.

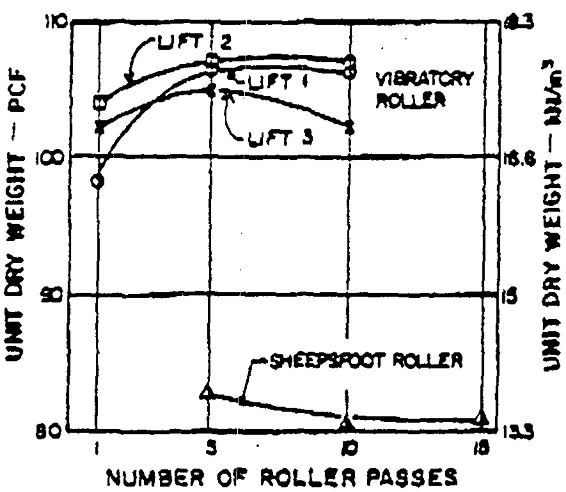

FIGURE 5
EFFECT OF ROLLER PASSES ON COMPACTED DENSITY

For a comparison with compacted areas, 37 field density tests were performed on uncompacted site landfill deposits. The lowest unit dry weights were found to range from 7 to 8 kN/m^3 for a cinder and ash. The typical unit dry weight of landfill ranged from 11.7 to 14.2 kN/m^3. Based upon these unit weights, a volumetric shrinkage of 17% was calculated.

Earthwork operations started in September, 1975. Field density tests were performed on each lift of soil. The compacted densities were found to range from 90 to 95% of 17.8 kN/m^3. Fortunately, the cut and fill operations occurred above the water table and the water content of the landfill was near optimum for compaction. Some blading and drying was undertaken prior to compaction. Discing could not be undertaken because the large debris in the landfill frequently broke the discs. Where cuts were made below the water table in the transition area, the fines in the landfill made the fill unworkable. It was necessary to stockpile the fill to allow it to dry before compaction.

During November, 1975, wet and cold weather conditions developed and the degree of compaction declined to less than 90%. Since the winter months were approaching and less favorable conditions were anticipated, the remaining fill was stabilized with western coal flyash containing a high lime content. The flyash was spread from trucks and mixed by blading. The flyash content ranged from 6 to 10% by weight and this was sufficient to reduce the water content by about 4 to 6% prior to compaction. With this procedure, the compacted densities once again exceeded 90% of 17.8 kN/m^3. The following year, it was observed that some cementation occurred as a result of this stabilization. Approximately the uppermost 6 m of the fill was stabilized with the flyash.

It was originally anticipated that some of the landfill from the cut areas would be unsuitable for fill, either because of high organic content or large debris. However, all the landfill was found acceptable so none was removed from the site.

Following completion of the earthwork operations on November 22, 1975, 12 settlement observation plates were embedded in the landfill where the thickness of compacted landfill ranged from 9 to 18 m to measure post-construction settlement. Elevation readings were taken four times per month until February, 1976. The majority of the settlement occurred within the first month following completion of filling, and almost no settlement occurred after January 8, 1976.

In the building areas where 95% compaction was achieved, the post-construction settlements were on the order of .4 to .5% of the height of the fill. In parking areas, the degree of compaction was relaxed to 90% and the settlements were on the order of .7 to .9% of the height of fill. These compressions are typical for granular deposits (3).

DYNAMIC COMPACTION

Dynamic compaction was undertaken using a 6-tonne weight dropped from a height of 11 to 12 m. In the compacted fill areas, the dynamic compaction was applied only at the footing locations to further improve the deposits. In the cut and transitional areas, dynamic compaction was undertaken on a grid basis throughout the entire building area plus 3 m beyond with a 2-m spacing between the center of the impacts. This was followed by additional impacting at the footing locations. The applied

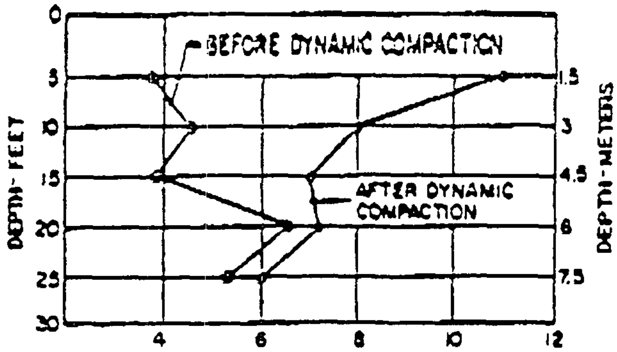

FIGURE. 6

INCREASE IN LIMIT PRESSURE DUE TO DYNAMIC COMPACTION

energy ranged from 130 ton-meters/meter^2 (Tm/m^2) in the slab area to 260 Tm/m^2 in the footing areas. Details of the dynamic compaction were presented in an earlier paper (4).

Soil borings and pressuremeter tests were completed following dynamic compaction to determine the degree and depth of improvement. The results of the pressuremeter tests at a typical footing location are summarized in Figure 6. In this area, there was a cut of about 8 to 10 meters prior to dynamic compaction. Improvements to depths of about 6 m were noted. This was considered to be a satisfactory depth improvement since the deep seated deposits had been previously densified by the landfill that was removed from these areas.

Typically, limit pressures of 5 to 10 bars and modulus values of 50 to 100 bars were achieved within the depth range of 1 to 6 m below grade. Typical limit pressure test results are shown on Figure 3. The limit pressures following dynamic compaction or roller compaction were not as high as measured in the test section where the the limit pressures ranged from 11 to 13 bars and the pressuremeter modulus ranged from 120 to 130 bars. This is attributed to the lower degree of compaction in the mass fill than was attained in the test section. However, a minimum limit pressure of 5 bars or modulus of 50 bars was considered acceptable for this project to produce the proper bearing capacity and limit settlement to tolerable values.

At a few isolated locations, the landfill was so weak or wet that the weight would become buried below the landfill surface following impact. At these locations, crushed stone fill was deposited within the craters and dynamic compaction resumed until satisfactory resistance was obtained. Weaker-than-normal support conditions were also encountered at the boundaries of the landfill and natural clay deposits. Some arching of the landfill may have occurred, thereby resulting in a looser condition of the landfill immediately adjacent to the near-vertical faces of the clay deposit. At all but a few isolated locations, it was not necessary to place granular fill at the surface of the landfill to provide a mat for the weight to impact into. Following dynamic compaction, the landfill was levelled by pushing the fill from between the craters into the craters. The surface area was then densified by making three passes with a fully loaded dump truck.

In the cut and transition areas, the average ground depressions following dynamic compaction ranged from 30 to 50 cm. This ranged from about 5 to 10% of the thickness of the deposit that was densified. In the fill area where flyash stabilization and conventional compaction was undertaken, the crater depths were only on the order of 20 cm and the average ground depression only about 7 cm. This is attributed to the cementation that occurred within the landfill from the flyash as well as from higher compacted densities in the fill.

PERFORMANCE

Settlement readings were taken on footing foundations in both one and two-story buildings as the structures were constructed and for one to two months thereafter. Measured settlements were on the order of 1.5 to 2 cm which was the magnitude predicted based upon an anticipated pressuremeter modulus of 50 to 60 bars that would be achieved following dynamic compaction or conventional compaction.

CONCLUSIONS

After the organic material has decomposed, landfill deposits can be classified as suitable materials for engineered construction. At this site, the landfill behaved similar to a granular soil with fines and a slight cohesion.

The landfill deposits at this site were compacted by different methods. In fill areas, thin lifts of controlled fill were compacted by conventional compaction equipment consisting of a vibrating roller. Compacted densities ranging from 90 to 98% of 17.8 kN/m^3 were achieved.

In cut and transitional areas, the landfill deposits were compacted by means of dynamic compaction. Pressuremeter modulus values ranging from 50 to 100 bars and limit pressures ranging from 5 to 10 bars were achieved within the zone of improvement.

Both methods of densification were sufficient to limit settlement for one to two-story buildings to values on the order of 1.5 to 2 cm.

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3) The common practice of slurry control testing appears to be insufficient to detect in-trench slurry problems that may arise quickly when a localized zone of variable groundwater chemistry is encountered. However, continued use of fluid loss testing (as well as viscosity and density tests) is advisable to detect undesireable variations in bentonite quality.

4) In order to design future quality control/assurance programs, backfill testing for consistency (slump and moisture content) and fines content may be reasonably expected to result in a normal frequency distribution of test data. Such an assumption regarding common slurry control testing does not appear to be warranted.

Our experience on this project indicates, above all, that the key to successful slurry trench construction is the experience, skill, and conscientious diligence of the principal personnel on the project. Because the trench excavation is performed entirely below the level of the slurry in the trench and out of the equipment operator's view, the skill of the backhoe or clamshell operator is a key factor. The individual responsible for slurry quality is similarly of primary importance. Finally, the contractor's superintendent, owner's field representative and construction inspectors must all diligently observe and understand each facet of this complex geotechnical construction procedure in order to assure its successful completion.

ACKNOWLEDGEMENTS

Special appreciation must be given to the City of Pontiac for graciously allowing the presentation of this paper. In addition, Mr. Sharmyn Elliott, our field representative on the project, deserves special thanks for diligently monitoring all facets of this project and making this compilation possible.

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LANDFILL STABILIZATION FOR STRUCTURAL PURPOSES

BY

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ABSTRACT

The insitu stabilization of landfills and waste disposal sites for structural and environmental purposes has recently been accomplished through the application of existing soil stabilization and ground modification technologies. This paper discusses landfill stabilization projects which have utilized two existing methods, pressure injection stabilization and dynamic compaction. The technologies for lime/fly ash (L/FA) slurry injection and dynamic deep compaction (DDC) are discussed and job photographs of contractors injection and compaction equipment on actual landfill closure construction sites are shown and case histories of injection stabilization and dynamic compaction of sanitary landfill solid waste disposal sites are presented. The use of lime slurry pressure injection (LSPI) for control of methane gas is discussed with note of pH values required for methane gas control. An interesting solution to excess landfill subsurface water or leachate is presented whereby it is proposed to use leachate for the mixing of L/FA slurry grout to be reinjected into the site, thus cementing the unwanted liquids permanently into the waste disposal site. Included in the paper is information concerning a recent laboratory lysimeter test program using lime slurry and lime/fly ash slurry pressure injection conducted by the U.S. EPA at the University of Cincinnati. Structural calculations for insitu strength and settlement considerations are included for the closure of a landfill site to be used for building or road construction. A proposed new combination landfill stabilization method using dynamic deep compaction followed by LSPI and L/FA slurry injection is discussed. The need for new research data to support structural landfill closure is recommended.

INTRODUCTION

Grouting and compacting a landfill mass to reduce settlement is perhaps the one facet of the general subject of geotechnical practice for waste disposal that is thought to be non-environmentally related. It is generally understood that there are compelling engineering design reasons for strengthening existing landfills to allow for construction of foundations for new buildings and roads, but what is

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not general knowledge is that an unstable landfill mass increases the potential environmental hazards. Leachate fluids and gases are much more likely to be emitted from structurally unstable landfill masses than from strong, dense, stable, compacted and treated landfill masses. It is actually difficult to improve a landfill mass structurally without at the same time generating beneficial environmental side effects by causing improvements in present and future leachate and gas generation. This paper will touch briefly on these environmental considerations of landfill stabilization; however, as cited in the references, experience has shown that environmental improvements are also often tied directly to structural grouting and compaction practices, (Blacklock, Josi, Wright, 1982).

The case histories of recent landfill stabilization projects cited in the references reflect the existing state-of-the-art technology for grouting and compaction of landfills utilizing both Lime/Fly Ash (L/FA) slurry injection and Deep Dynamic Compaction (DDC). This paper provides a review of these two methods of stabilization. It should be noted that these methods represent the development of relatively recent technologies for landfill stabilization. Prior to their development, the principal methods of dealing with landfill construction sites were to either excavate the garbage and transport it away or to surface compact and add fill of compacted rock and soil prior to construction. Experience has shown that these methods should not be recommended for long term solution of landfill construction sites, since excavating garbage generates new hazards to the environment, and surface compaction and fill can cause the roads or buildings constructed on them to structurally and environmentally fail. With today's ever-increasing environmental awareness, the hazards of the first are obvious, whereas the hazards and shortcomings of the second have become visible only through documentation of failed buildings and roads and condemnations of occupied buildings due to methane gas. There have been many well publicized failures from building on unstabilized landfills as well as less publicized subsurface leachate flow problems that have been created when landfills were not properly stabilized.

One suggestion for a better system for stabilization of existing landfills is to treat the structural foundation problem and the environmental leachate and gas generation problems as one. To that end, this paper discusses the application of the methods of L/FA stabilization and DDC separately and in combination for landfill treatment. It is believed that with the combination of both systems the end result will be structurally and environmentally better than with either system used independently. To date, the first L/FA - DDC project has yet to be funded; however, several are in the planning stages and it is likely that one will be completed in the near future. Meanwhile, it would be well to plan new research and development projects to generate long term data for the L/FA - DDC method of enviro/structural treatment of existing landfill masses.

DISCUSSION

In many areas of the nation, particularly near large cities, solid waste landfill capacity and available new landfill sites are declining. One way that this situation might be improved is to reopen closed landfills and expand them vertically. To accomplish this, old landfills must be stabilized to accommodate the added mass and its leachate handling system. Also, as land for commercial and industrial development near population centers becomes more valuable, stabilization of old landfills becomes increasingly attractive. One method to stabilize a solid waste landfill is to inject cementitious grout materials into the waste mass to fill voids and increase strength (See Fig. 1). Lime and fly ash are two relatively inexpensive materials that have been utilized for that purpose. The Lime/Fly Ash (L/FA) injection stabilization process is a patented process owned by Woodbine Corporation of Fort Worth, Texas. The Lime Slurry Pressure Injection (LSPI) method of soil stabilization was developed in the 1960's for in-situ stabilization of expansive and low strength clay soils for stabilization of buildings, highways, railroads, embankments, fills and slopes.

In recent years the additive, fly ash, has enjoyed increased use in lime injection stabilization. The process of pressure injection of lime slurry mixed with fly ash has been termed lime/fly ash (L/FA) injection, (Wright, 1978). Lime/fly ash slurry was initially noted to result in a more pronounced increase in the bearing strength of silty and sandy soils than lime slurry alone. With many well drained soils deficient in reactive minerals, lime slurry alone is usually not effective for increasing strength but with a proper mixture of lime and fly ash into a groutable slurry, injection stabilization can be successfully extended to non-reactive soil types. The use of L/FA injection for landfill stabilization quickly followed. An advantage of L/FA injection over cement grouting for landfills is that fly ash, being a by-product, is relatively inexpensive, about 25-50 percent of the cost of cement. Therefore, it offers an economical solution by providing a low cost slurry capable of developing strengths up to 1000 psi or greater. This economy is especially important for projects with large void ratios such as municipal landfills and other large waste disposal sites.

Another use for lime/fly ash slurry has been for the renovation and releveling of failed concrete pavements and foundation slabs built over unstable landfills (See Fig. 2). Pressure injection increases the strength of landfill foundations by adding tensile reinforcing strength, mending existing cracks and filling voids causing strength of the clay cover, embankment fill and the foundation subsoil to be increased simultaneously, thus reducing settlement and stopping progressive failures. Crack mending is critical to waste site renovation, since landfill strength may not be satisfactory if the landfill, cover and subsoil are cracked. Cracks may develop because of excessive tensile stresses due to differential settlements or because of consolidation under surcharge or shrinkage due to material decay and drying. Many tension cracks frequently begin in the bottom layers and may not be detected until the foundation is already failing. The pressure injection method has been developed to treat cracks and planes of weakness insitu, even those hidden from view that start at the bottom. Surface repair does not mend deep existing tension cracks

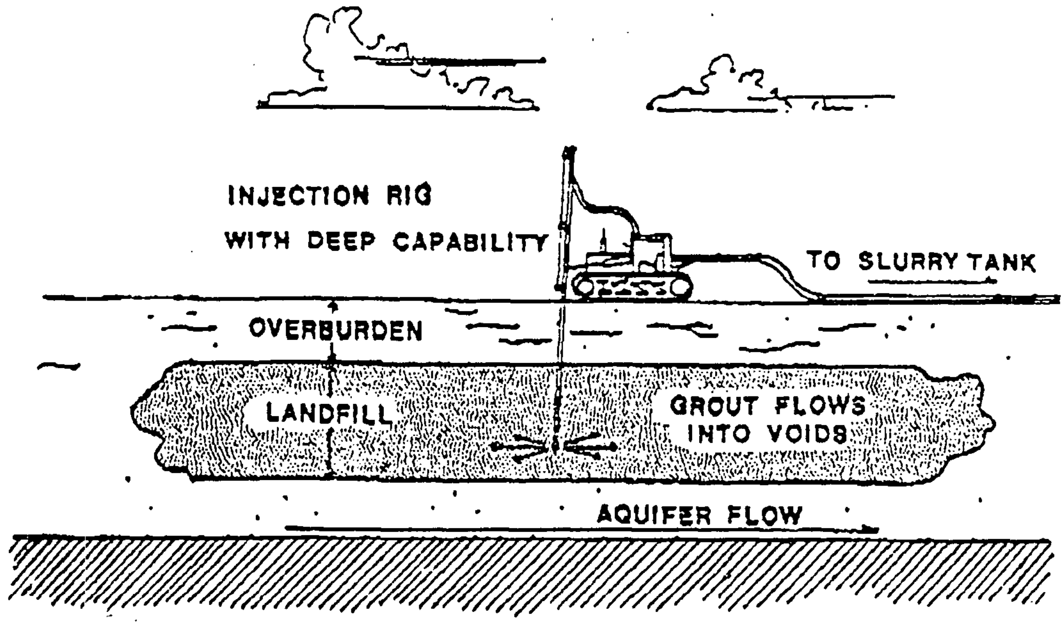

Fig. 1 - Landfill Injection Stabilization Concept

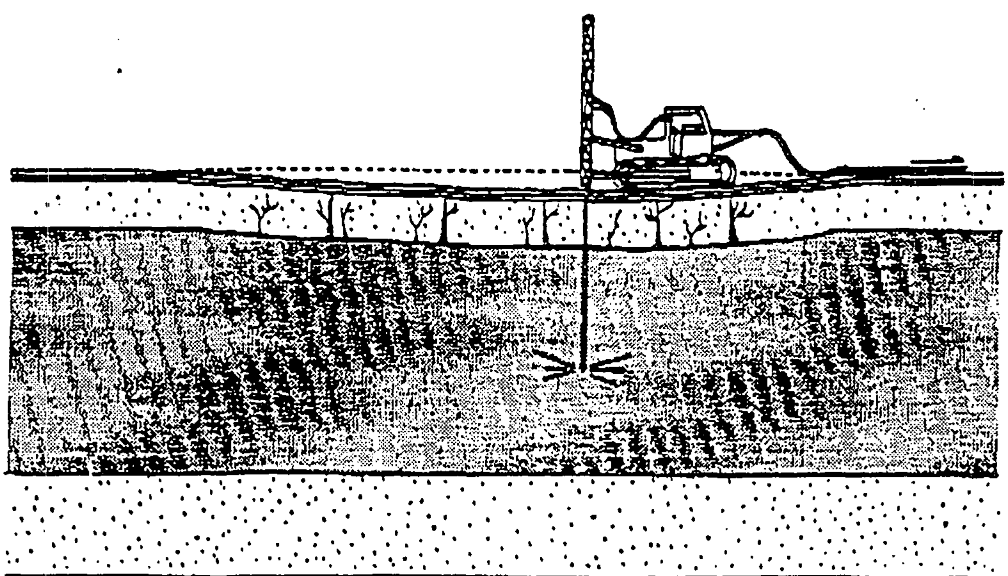

Fig. 2 - Highway Renovation with Injection Stabilization

in the undisturbed mass, and the cracks will continue to propagate, causing long term failures unless repaired.

Another field of application for pressure injection is in the treatment of toxic wastes. Lime is well known for its ability to "fix" heavy metals, and the high alkaline environment made possible by injecting lime slurry can inhibit mobilization of heavy metals and other contaminants. A recent evaluation of various methods of remedial action of uranium mill tailings concluded that L/FA injection could offer a viable solution (Tamura and Boegly, 1982).

DESCRIPTION OF THE LIME/FLY ASH PRESSURE INJECTION PROCESS

The Lime/Fly Ash (L/FA) pressure injection process consists of pumping into the ground a slurry mixture of hydrated lime, fly ash and water containing from 30 - 60% dry solids. Injections are made vertically into the ground with holes typically spaced on a grid pattern. Initial injections are often followed by secondary or even tertiary injections, spaced equally between the previous injections. Depth of injection will vary based on specific job site conditions, e.g., typical depths 3-m to 12-m for landfills. Typical mobile injection units used for stabilization work up to 6-m deep. A standard injection rig is equipped with three or four injection pipes that can be hydraulically pushed into the soil. The pointed tip of each injection pipe has a perforated hole pattern which disperses the slurry in a 360 degree pattern throughout the depth of injection.

The L/FA slurry pressure and flow are obtained from a large displacement type pump, which is mounted on the slurry mixing tank, which is equipped with a mechanical agitator and is capable of bulk mixing 28 - 40 tonne of lime/fly ash. The resultant L/FA slurry is pumped at pressures of 350 to 1400-kPa through a high pressure hose to the injection rig. Slurry is injected at frequent depth intervals to refusal or in a slow continuous motion either up stage or down stage until a specified quantity is injected. The slurry, following the paths of least resistance, is forced laterally and vertically into voids, cracks, and fissures, and available voids to form honeycomb of L/FA grout throughout the landfill mass. Normally it is necessary to make secondary or tertiary injections to install a sufficient quantity of L/FA grout into the landfill mass. The subsequent injections are spaced equally between previous injection holes and are pumped to slurry refusal or until a predetermined quantity is injected. The resulting L/FA grout seams become moisture barriers that impede moisture movement and add tensile reinforcement and compressive strength throughout the stabilized mass.

The amount of L/FA slurry required for stabilization can vary considerably, depending on L/FA grout material properties, injection depth and spacing, permeability of the mass and degree of strength and stability required. A typical value of slurry required is in the range of 20 to 40 kg per cu m for a single injection and about 60 to 80 kg for a double injection.

When stabilizing existing landfills that are full of water and

leachate, the question of how to dispose of the displaced water represents a difficult construction design decision. One solution that has been proposed is to establish well points ahead of the injection operation to extract leachate by recycling into subsequent L/FA slurry and thus placed back into the ground as an integral part of the stabilization process. Even highly acidic leachate can be neutralized with available lime at the site and reused for injection. In a project using L/FA and DDC the liquid could be stabilized prior to the DDC. This would be more difficult, but DDC cannot be used on a saturated site.

In remedial landfill L/FA injection applications a substantial portion of the voids should be filled to achieve adequate bearing capacity and stability to existing structures. L/FA injection is often a feasible void filling method in conjunction with stabilizing the actual landfill materials. Should the injected lime/fly ash slurry subsequently crack, it possesses the inherent ability to reknit the cracks due to a phenomenon called autogeneous healing that also occurs in lime-based mortars. Lime/fly ash slurries harden as a function of time and temperature, but generally less rapidly than Portland cement grouts.

In some projects it is desirable to utilize a combination of both lime and lime/fly ash pressure injection in stages especially where excessive acidity is present or where primary treatment is necessary for methane gas. A 48 hour curing period is allowed between successive injections. LSPI or L/FA pressure injection should never be applied in freezing weather, and should not be done at 5 C and falling temperature. When temperatures are marginal, the elevated temperature of hot slurry made by job site slaking quicklime into hydrated lime slurry is an advantage, (Boynton and Blacklock, 1985).

Both lime and lime/fly ash slurry mixtures are equally viable for deep insitu landfill treatment with modern pressure injection technology and the choice of how much and which material to use is an engineering decision based on individual job site conditions as determined by field investigations and laboratory tests. Usually, multiple injections are necessary to achieve full stabilization for more concentrated treatment. In some instances joint use of lime and lime/fly ash pressure injection in stages, with a curing time between applications, is the indicated method to employ for full permanent enviro/structural closure.

DYNAMIC DEEP COMPACTION

Dynamic Deep Compaction (DDC) is a ground modification process for increasing the stability and strength of landfills for support of shallow foundations for buildings and roads. It involves the application of very high energy impacts on the surface from heavy 9 - 18 tonne weights (See Fig. 3 and 4). The dynamic impact of a heavy weight, dropped from heights up to 30-m, tranmits shock waves downward through the rock cover and the deep landfill layers which compacts and re-arranges them into a denser stronger configuration. DDC reduces the permeability, porosity, and volume of the voids, thus reducing subsurface flow and increaseing the strength. It does not treat pH levels to reduce methane gas generation; however, it does typically reduce leachate generation through reduction of ponding caused by long term settlement and restriction of free water flow. In some applications DDC can be used independently and in others it should perform best when combined with LSPI or L/FA injection stabilization to further increase strength, reduce settlement, reduce methane gas generation and impede leachate migration. Successful site improvement using DDC involves:

1) Accurate predictions of energy and drop-spacing requirements.
2) Careful and continuous control of operations at the job site.
3) Knowledgeable geotechnical testing to verify effectiveness.

To date, most applications for landfill stabilization have been for the purpose of highway construction; however, a review of many projects in the planning stages indicates that this proven economical system is a likely candidate for enviro/structural waste site closures and remedial restorations.

LANDFILL STRENGTH CALCULATIONS

The calculation and prediction of strength in L/FA injected landfills is based somewhat on conjecture and appraisal of the original landfill contents; however, there are certain known strength facts concerning the compressive strength of neat L/FA mixtures which can be used. Two hardened L/FA grouts with moisture contents of 50 and 80 percent have been selected to represent the high and low grout strengths anticipated in an average landfill injection project. These two grouts represent a compressive strength spread of from 14 to 3.5 MPa respectively. The resulting compressive strengths obtainable in an injected landfill due to the increase in strength resulting from the hardened injected grout is shown plotted (See Fig. 5). Experience has shown that injection can result in typically placing 70 kg of solids per cubic metre of treated landfill mass. From the curves it can be seen that this could result in a high vertical bearing strength value of 690 kPa if the hardened grout is formed at 50 percent moisture content or a low strength value of 220 kPa if the hardened grout is formed at 80 percent moisture content. The major factors effecting the moisture content of the hardened grout include available water in the landfill, density of the landfill and total amount of slurry injected. While it is difficult to determine quantitively the final strength, it should be within the range of the two curves shown. (The high curve is for 13 mPa grout and the low curve is for 3.5 mPa grout.) Intermediate values lie between the two curves.

ENVIRONMENTAL LANDFILL STABILIZATION APPLICATIONS OF LSPI AND L/FA

In the future, the potential for air quality and related health impact from landfill gas (LFG) emissions will be scrutinized carefully by the public and regulatory agencies. Conceivably, LFG emissions control could become as important as leachate control at our landfill disposal sites (Stearns and Beizer, 1985). LFG is generated by

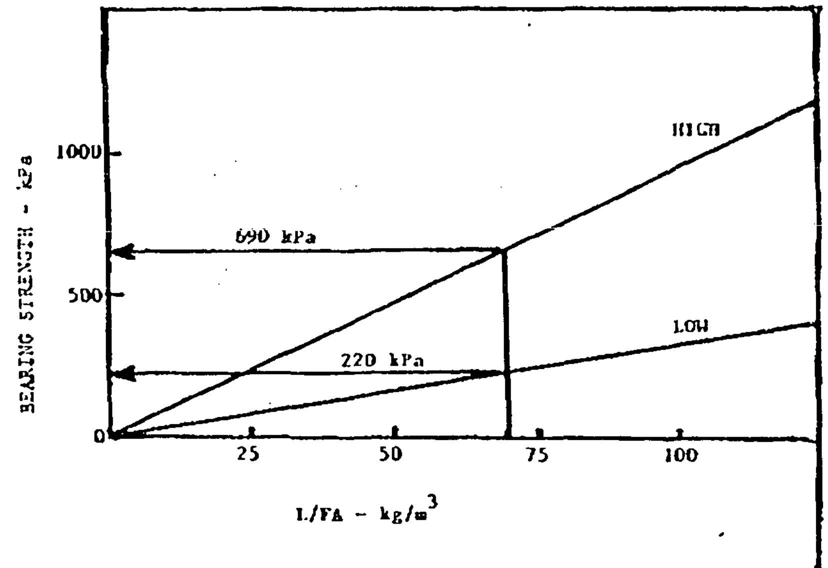

Fig. 5 - Landfill Bearing Strength Vs. Weight Dry L/FA Injected

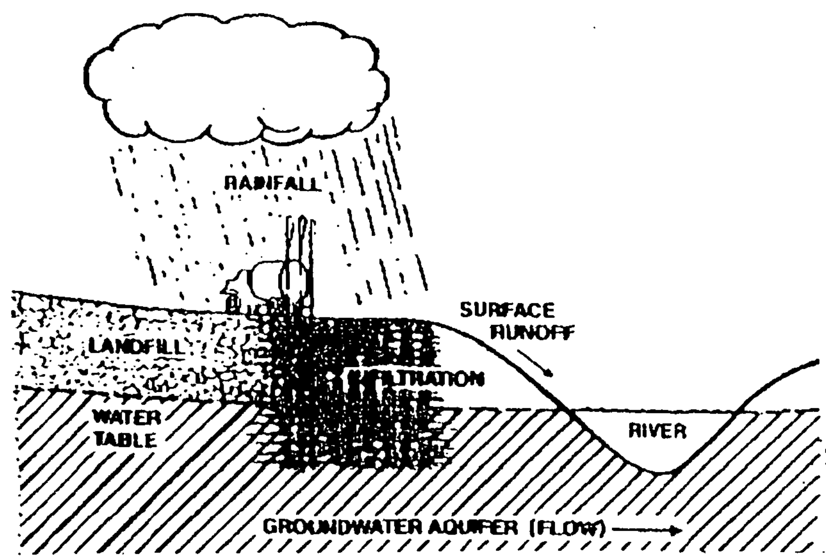

Fig. 6 - Sketch Showing Environmental Injection Benefits for Leachate Problems

natural anaerobic decomposition of organic waste within the landfill. The LFG is composed primarily of methane and carbon dioxide. For years, the principal concern of regulators and landfill managers has been the potential flammability and explosive hazard of the methane gas. The two basic approaches to the control and prevention of methane buildup and migration in municipal landfills have been installations of impervious barriers and gas ventilation systems; however, it has long been noted that LFG production is also pH dependent, especially with regard to increasing gas generation commercially. Methanogenic bacteria function best in the range pH 6.4 to 7.4. Calcium hydroxide has been often used to raise the pH into this range where methanogenic bacteria operate best to achieve greater methane gas production and in a few cases into an even higher range where methane gas generation is reduced or eliminated. Most refuse that is buried decays relatively slow, but by controlling the conditions such as pH and moisture, a landfill can be changed into a more effective decomposition site and a better gas production facility (Malone, 1984).

The concept of treating leachate movement with structural stabilization methods involves the reduction of flow characteristics of the entire landfill mass. A well compacted, well grouted landfill will be less likely to cause leachate problems, even when unfortunately it has been located near a river or over a ground water aquifer (See Fig. 6). As reported, (Blacklock and Wright, 1984) perculation test before and after L/FA injection have shown substantial reductions in water flow. More research needs to be done in this area, but it certainly looks very promising to date.

A method for the analysis of refuse stability was presented by Bookter and Ham, 1982. They reported data obtained by testing and analyzing refuse from test lysimeters and actual landfills across the United States. The rate of refuse stabilization in a landfill is a valuable parameter in predicting future leachate generation, gas production, and differential settlement. Of particular interest was the range of pH values found during their study in controlled-aged test lysimeters. The average pH found in fresh and one year test lysimeters was 6.9 and it was pH 4.4 at six years and pH 5.3 at nine years. In addition, the paper presented data showing the Los Angeles Area Landfill with a pH 5.3 - 7.5, the New York Area Landfill with a pH 8.9 - 9.1, and the Chicago Area Landfill with a pH 6.3 - 8.1. Clearly, this data indicates the complexity of the problem of reducing methane gas generation by pH control. Although this method has apparently worked in the case studies, (Blacklock and Wright, 1982) there needs to be much more study prior to general application of the LSPI and L/FA methods for this purpose.

One attempt to generate new data for greater understanding of methane reduction possibilities was funded by the National Lime Association (NLA) through Woodbine Corporation in 1985. A short test and research program was conducted utilizing six existing refuse filled lysimeters located at the University of Cincinnati and owned by the U. S. Environmental Protection Agency. In August, 1985, three test lysimeters were injected with lime slurry, two were injected with L/FA slurry and one was left untreated (See Fig. 7 and 8). The units were then monitored for methane gas production for five months, Sept. 1985 - Jan. 1986. As expected, the pH in each unit went up after injection; however, the average change was perhaps too small to produce the best benefits. The average pre-injection pH of the treated lysimeters was 5.7 and the final post-injection pH was 6.7 at the end of six months. In the month immediately after injection, the methane gas generation rose sharply from an average value of 907 L to 1872 L per month. Whether this increase was caused by the influx of new calcium rich moisture, it is not known for sure; but since the test lysimeters were already saturated and producing, it is most likely that the increase came solely from the raise in pH. At the end of six months the methane production value had fallen to 726 L per month. In each lysimeter there was a large increase the first month and a steady reduction each month thereafter. In this respect the data was consistent. In 1984, at the end of the previous test program, these same lysimeters had an average monthly production of 1427 L. The NLA test, although short and economically funded, indicated that the gas generated in the sixth month after treatment was 80 percent of that immediately prior to the test, and 50 percent of that which had stabilized as the average monthly production in the year 1984, prior to the test. Based on the success or lack of success of this short test program, depending on your point of view, the U. S. EPA Office of Research and Development proceeded in 1986 with plans for a full scale landfill L/FA injection stabilization program to include the development of gas, leachate, strength, and settlement research data. Unfortunately, the funding was withdrawn from this project prior to actual start of work. The complete research plans are on file and available when additional funds are made available to continue the project.

The many desirable characteristics of lime for stabilization of landfills also include treating spent pickle liquor impoundments by LSPI to raise the pH to 6.5 and then sending it into the city system, (Crowley, Brown and Anderson, 1984). In this project, two impoundments containing sludge and pickle liquor were treated with lime injection. The $Ca(OH)_2$ slurry was injected at 1.5-m intervals, using a small tractor equipped with 3.7-m injection rods. Closure of the active impoundments consisted of five phases:

1) Neutralization of the liquid and pumping to the city.
2) Neutralization and dewatering of the sludges.
3) Investigation of the soils beneath the impoundments.
4) Placement of a cover in accordance with regulations to close impoundments as landfills.
5) Continuation of ground water monitoring.

The clay cover was covered with topsoil and planted with grass. Ground water monitoring was planned to continue until all values stabilize or return to background levels.

SITE EVALUATION

Waste Site Investigation--An effective waste site investigation

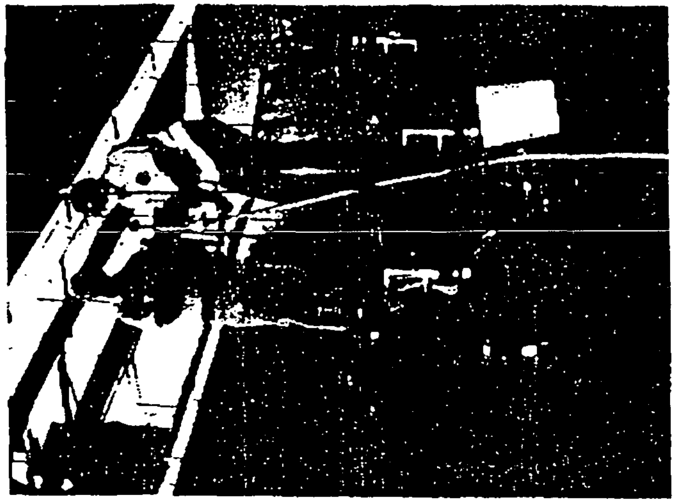

Fig. 8 - Lysimeter Injection in Progress

Fig. 7 - EPA Laboratory Experiment for L/FA Injection

should consist of a thorough surface and subsurface effort. The surface efforts should include examination of arial photography, topography and construction maps to pin point location of nearby physical features, wells, creeks, rivers, drainage ditches and surface gradients. In addition, the geotechnical engineers', owners' and contractors' representatives should meet at the site as necessary to walk out and plan the subsurface drilling program and later to plan the *enviro/structural stabilization program.*

The geotechnical engineer with associated geohydrologists and environmental engineers should conduct a thorough deep drilling and sampling program to characterize the layers and placement of refuse and collect samples for analysis. Laboratory analysis of the subsurface materials should include determination of information on soluble sulfates, total sulfates, pH values, metal concentrations, percentage of H_2O, percentage of volatile, percentage of cellulose, percentage of lignin, and date of placement. In some locations it might also be advantageous to perform before and after standard insitu geotechnical tests to determine the actual amount of improvement and the depth to which improvement was effective. These tests include standard penetration, pressure meter and dynamic cone penetration tests; and they should be used across the entire site and for the full thickness of the loose fill.

Material Test--In addition to the waste site investigation discussed above, it is also necessary to test all source materials for injection. It is well established that there is a considerable variation in fly ash materials from different sources and new tests are necessary for each project. Experience and chemical composition tests will help evaluate these performance properties; however, it is always best to evaluate the materials *structurally by performing* a series of cube tests or compression cylinder tests. These tests should evaluate time, temperature and strength variables for different mixing times, different mix ratios and different material manufacturers or sources. Where possible the test should utilize actual water samples from the landfill site to manufacture laboratory test samples.

Field Pump Tests--In many instances it might be advantageous to conduct a trial pump test during the design stage to determine the slurry volume placed with a single or double injection. Also it might be desirable to dig a trench to observe slurry flow in the trench side walls, especially if there is a question about available fissures and openings in the landfill mass to accept the slurry. This type of data also could be obtained from soil drilling equipment to depths extending to the bottom of the site.

Surcharge Tests--For certain sites where consolidation and settlement are the major structural problem, it could be advantageous to inject a test pad and then surcharge the pad as well as a control section and monitor the results. This has been used effectively in the past to evaluate the LSPI and L/FA systems at prospective waste site locations. Other methods of evaluation include percolation tests (Blacklock and Wright, 1984).

CASE HISTORIES OF LANDFILL ENVIRO/STRUCTURAL STABILIZATION

The two case histories that follow represent the state of the art of L/FA injection and DDC landfill stabilization. Many other case histories have been presented in the references, (Blacklock, Josi and Wright, 1982, Blacklock and Wright, 1984). These two were chosen to illustrate the recent experiences of owners, contractors and engineers with development of this new technology. Other recent landfill case histories include L/FA injection of a new constructed health clinic in Dallas, DDC of a new highway in Oklahoma, L/FA injection of low level radioactive waste at Oak Ridge and cement grouting of a failed occupied building site in San Antonio. Each of these cases as well as several others in the planning stages could serve to illustrate still other important facets of the enviro/structural technology for landfill stabilization.

Case History No. 1
Dynamic Deep Compaction of a Sanitary Landfill to Support Highway Relocation in Arkansas

Geotechnical exploration by the Arkansas Highway and Transportation Dept. uncovered a 170,000 m sanitary landfill closed in 1979 beneath the proposed right of way of a new four-lane divided highway north of Fayetteville, Arkansas, U.S.A. (Welsh, 1983). After determining that relocation was impossible and removal environmentally impractical, the Department's engineers decided to use Dynamic Deep Compaction (DDC) to densify the 10 m deep landfill which was installed in 1977 and 1978. The Department entered into a contract with GKN - Hayward Baker Company for the DDC portion of the construction. The Federal Highway Department provided 75% funding of the construction and 100% for the monitoring of the finished project.

A large modified crane dropped an 18 tonne weight from up to 28 m using three passes with an average of ten blows per pass. The entire site was depressed between 1.6 and 2.5 m. Due to the non-homogeneous nature of the fill, conventional geotechnical instrumentation was judged to be unsatisfactory to determine the effectiveness of the compaction system; therefore, two instrumented full scale load tests were performed and 30 permanent settlement plates were installed.

The DDC work performed at this site created a stiff rock mat some 3 m thick and resulted in a net compression of the landfill some 1.6 to 2.5 m or 20 to 25 percent of its original depth. The trash beneath the rock mat was substantially compacted by the DDC method. The road was completed and open to traffic in December 1984. As anticipated, the fill material has continued to slowly settle due to decomposition of the organic constituents. In 1985 the AHTD reported one major area of settlement; however, overall, the settlement was not generally deemed noticeable at highway speeds and no corrective measures were necessary at that time. Monitoring is planned to be continued.

Case History No. 2
San Antonio Municipal Landfill Case History

In the spring of 1986, a municipal landfill building site for a new reinforced concrete tilt-up wall office/warehouse was stabilized with L/FA injection. The waste refuse material was determined by a geotechnical investigation to be 5-m deep with 2 m of expansive clay cover cap. The landfill which had been closed for several years was located near the airport in a prime commercial area. The building site was 8600 sq. m, of which 3000 sq. m was to be under the building. The walls and floor of the building were to be supported on drilled piers to 6-m deep. The owner and developer were concerned about the bearing strength for the parking lot surrounding the building, the expansive movement of the clay cover, the long term settlement and the possibility of methane gas generation of the entire site. Based on the prior success of the L/FA method in treating similar sites, it was recommended to double inject the site with L/FA slurry to a depth of 5-m over the entire site and to LSPI stabilize the clay cap to 2-m under the footprint of the office building. The LSPI treatment was later omitted due to schedule and economic considerations.

The contractor injected the site using three slurry mixing tanks, one for quick-lime slaking and two for continuous mixing of L/FA slurry. The slurry was injected with trac-injectors with 6-m injectors. The total amount of dry lime/fly ash material installed was approximately 1600 tonnes. At the conclusion of the stabilization the site was leveled, exposing a hard thick seam of L/FA grout in the clay cap. The piers were drilled next and then cased and concrete installed. The building was designed with a dock high floor utilizing numerous rectangular vents as a precaution in the event that a methane gas problem developed. There are no plans at this time to continuously monitor the site; however, it will serve well for a long term study since the construction was well documented by the contractor, Woodbine Corporation, and the developer.

SUMMARY

Based on several individual case history experiences of LSPI, L/FA and DDC, it is believed that a proposed combination stabilization method of LSPI, L/FA and DDC has much future potential for economical treating of municipal landfills. Today there is much concern over toxic leachates contaminating of ground water aquifers, and it appears that LSPI and L/FA could play a major role in protecting ground water, neutralizing leachate plumes, and for placing curtain walls to prevent leachate migration.

From the foregoing discussion and case histories of landfill stabilization experience, it has been shown that lime and lime/fly ash pressure injection as well as deep dynamic compaction are promising approaches for waste site stabilization for both remedial closure and preventative purposes, and that added potential lies in the use of combining these two technologies where DDC would initially reduce large voids and L/FA grouting would strengthen and seal most remaining smaller voids. The diversity of applications to date includes building foundations, parking lots, highways and controlled hazardous waste sites, as well as acid neutralization and methane gas control. LSPI can be used as a single treatment method for pH control or in conjunction with L/FA injection and deep dynamic compaction where settlement and strength are factors. By using the correct combination of the three different technologies for landfill stabilization, LSPI, L/FA and DDC can be expected to reduce potential settlement, increase internal shear strength and surface bearing values, stabilize moisture content, cement free subsurface moisture, impede seepage and flow of leachates and reduce methane gas evolvement.

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APPENDIX 2

Example of Gas System Operating Plan

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Adrian Landfill Gas Collection/Flare System
Maintenance Program

Maintenance Supervisor   Robert Willis (Division Manager)

The landfill gas collection system consists of the landfill gas wells, the gas collection header piping and the condensate collection sumps. The gas flaring system consists of the inlet water knockout, two gas blowers, an air operated valve, the electrical controls and the gas flare.

The principal purpose of maintaining the gas system is to maintain flow to the flare so to prevent offsite gas migration. This will also prevent air flow into the refuse which causes an aerobic reaction that prevents the formation of methane. Air intrusion can also be the cause of subsurface fires which can cause the landfill surface to collapse and/or smoke to emanate from the landfill surface.

Maintenance of the gas system shall consist of a minimum of biweekly monitoring of the gas wells. Monitoring should include measurement of the methane and oxygen content and the wellhead vacuum. The methane percentage should be maintained between 40% and 65%. Oxygen levels should not exceed 2% by volume. Vacuum has no set value, but all wells should have some vacuum to maintain gas flow to the flare.

Adjustments to the wellhead vacuum should be made as follows:
If methane is below 40%, or if oxygen exceeds 2%, the vacuum should be reduced to prevent further oxygen intrusion into the refuse. Methane levels above 65% usually indicate that an insufficient amount of gas is being extracted. In this case, wellhead vacuum should be increased. If methane falls below 35%, the gas well valve should be closed fully. This will allow the refuse in the vicinity of the well to consume the entrained oxygen and, after time, become anaerobic. When the methane percentage exceeds 45%, the valve can be opened again.

A gas well monitoring log shall be maintained on site. This log should include the well number, the date tested, recorded values and a notation of any adjustments made.

The condensate drains in the gas piping allow the liquids in the system to flow to collection sumps. Condensate is then transferred to a holding tank for approved disposal. If the drains fail, condensate will build up in the gas piping and may cause blocking of the gas flow. This condition can be detected by observing the vacuum at the pipe header valves and at the inlet to the gas flaring station. Wide swings in vacuum, greater than 4" of water, usually indicate water buildup in the piping. If this condition occurs, condensate drains in the area of the vacuum fluctuation should be checked for proper operation. Header valve vacuum shall be recorded in the same log as wellhead monitoring data.

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Maintenance at the gas flare station shall consist of gas flow monitoring and periodic maintenance of equipment as described in the manufacturer's information.

The inlet conditions of the gas at the knock out scrubber shall be recorded as described for the gas wells. The scrubber itself should be examined weekly to be sure that water is not accumulating. This can be accomplished by examining the site glass on the side of the vessel.

The gas blowers shall be maintained as described in the "Lamson Corporation Product Data Installation & Operating Instructions". The blowers shall be lubricated after every two months of operation as described in the LUBRICATION Section of these instructions.

Only one gas blower need to operate at a time to maintain adequate gas flow off the landfill. To insure that both gas blowers are always capable of operation, the duty blower should be switched on a monthly basis. To do this, shutdown the system, switch the blower select button on the control panel to the opposite blower, close all valves to the present blower and open all valves to the opposite blower, then restart the system. System start procedures are described in the McGill Flare Manual.

Each blower has a vacuum gauge located at its inlet. A high level of vacuum indicates blockage upstream of the blower. If this condition exists, start the non-operating blower and shutdown the blower in question. After shutdown, examine the inlet screen for blockage.

Each gas blower is protected against low flow conditions which cause surge. Surge is where gas flows back and forth through the blower. This condition can cause severe damage to the blower impellers and bearings. It is usually caused by a pipe blockage upstream or downstream of the blower. Protection against surge is provided by an electrical control located near the flare control panel.

When low gas flow conditions occur, the blower motor amperage will fall below a set point. This will open a switch which shuts down the blower. A light on the front of the panel will indicate if surge occurred. [The meter on the front of the panel is a meter calibrated to gas flow. It only gives a magnitude of the flow and is not considered to be accurate, to better than 10%. The orifice plate should be used to determine an accurate gas flow.]

If the blower shuts down on surge, the cause of the pipe blockage needs to be determined. Restart the system and check inlet vacuum and discharge pressure. The high value will indicate which side of the blower the blockage is on. If it is in the inlet, check the inlet screen, water knockout scrubber or the condensate drain. If it is on the discharge, be sure the main inlet valve is operating properly and that there is no buildup on the flame arrestor.

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The main inlet valve at the flare opens after a pilot flame has been confirmed by the ultraviolet flame sensor. This valve remains open unless there is an interruption of the flame in the flare. If this occurs, the valve will automatically shut. This valve is held open by air pressure from the air receiver located next to it. No normal maintenance is required for this valve. The air pressure in the receiver shall be maintained at a minimum of 85 psig at all times.

The gas flare temperature should be maintained between 1400 degrees F and 2000 degrees F. Temperature adjustment is made by setting the controller to the desired temperature. The manual louver may have to be adjusted so that the automatic louver is not fully opened or closed. The McGill Flare Manual has detailed information on the operation of this equipment.

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Maintenance

- Monitor the gas wells and header valves biweekly for methane/oxygen percentage and vacuum.
- Maintain gas flow into the flare between 500 (min.) and 1500 (max.) cfm and methane percentage at 40% or greater.
- Switch the duty gas blower monthly.
- Lubricant blower bearings as described in LAMSON Manual.
- Maintain the air pressure in the receiver tank at a minimum of 85 psig.
- Check the level of propane in the storage tanks weekly. Keep at least one tank full at all times.
- Maintain flare temperature at 1400 degrees F or greater.
- Replace flare temperature recorder paper monthly.

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Malfunction

In the event that the gas flaring system experiences a breakdown; the following procedure shall be adhered to:

- Determine the cause of the breakdown. If the problem is minor and can be corrected, do so as soon as practical and restart the flare.

- If the breakdown exceeds two hours, call the DNR Air Quality Division at 517-788-9598 and notify them of the situation as soon as reasonably possible. The permit number for the flare is 799-89.

- Submit to the commission, in writing, within 10 days, a detailed report, including probable causes, duration of violation, remedial action taken, and what steps are being undertaken to prevent a reoccurrence. These preventative steps shall become a part of any malfunction abatement plan required by rule 911.

Shutdown:

A - Check to see if the gas blower shutdown on surge:

1) Restart the blower and flare and then determine inlet vacuum and discharge pressure.

2) Vacuum above 45 inches w.c. indicates plugged inlet:
   - Clean inlet screen.
   - Check water knockout and condensate drain for blockage.

3) If the discharge pressure is above 20 inches, it indicates a downstream valve is closed or that there is a buildup of material on the flame arrester:
   - Clean flame arrester by removing it and then steam jetting it clean.
   - Check the air pressure to the main inlet valve - this should be 85 psig or greater.

B - Check methane and oxygen quality of inlet gas.
   - Methane values should be between 40% and 65%.
   - Oxygen should be below 2%.

C - If flare has a shutdown on high temperature or pilot failure:
   - Check page 13 of McGill Flare Manual for troubleshooting procedure.